DESIGN, PLANS AND SPECIFICATIONS

FOR

THREE STORY REINFORCED CONCRETE MACHINE SHOP BUILDING

BY

H. S. ELLINGTON

Armour Institute of Technology

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DESIGN, PLANS AND SPECIFICATIONS

For

Three Story Reinforced-Concrete
Machine Shop Building
80'-0" x 120'-0"

A - THESIS,

Presented by

Harold S. Ellington

To the

PRESIDENT AND FACULTY

of

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-INTRODUCTION-

The author in attempting to introduce so broad a subject as "Theoretical Reinforced Concrete Designing" has found no little difficulty. Volumes have been written on Reinforced Concrete Designing and to condense the works of prominent writers in connection with a limited discussion of the various theories, proposed, has been the object in view. I have endeavored to discuss these theories intelligently so that the reader will be able to understand my reasons for pursuing the lines, I do, in the following discussion on the design of the Reinforced Concrete Building.

The theory of reinforced concrete has been the subject of much study by engineers and mathematicians for a number of years. Theoretical investigations in conjunction with practical tests have been made and much valuable data obtained, but, unfortunately, I have found very little uniformity in tests made by different experimenters and discrepancies in results necessarily appear, nevertheless, much knowledge has been obtained regarding the practical and theoretical solutions of problems involved in Concrete buildings, and such results will be incorporated in this work.

As the main object of reinforced concrete is to secure a material which will withstand strains due to transverse loading, it is of prime importance to secure a theoretical formula or formulas for use in the design of the section of beams,

girders, columns, and slabs at the point where the bending moment is a maximum. It is desirable, if possible, to secure a rational formula, but it is not absolutely essential to successful design that the formulas be rational, as empirical formulas, if properly applied, may, and do agree closely enough, for all practical purposes, with the results obtained from actual tests upon reinforced concrete pieces. Many such formulas are used in the design of reinforced concrete structures, yet, other things being equal, it is desirable to use the formula or formulas which embody most fully all conditions entering into the problem.

In order to make a more extensive research of theoretical designing, I have divided the Thesis into three chapters, each of which will have a definite object and simplify the work. Chapter one will contain the development of various theories with a limited discussion upon each. Those theories chosen are the most widely used in this country at the present time and I will be better fitted to argue for the one used in designing my building after discussing each.

Following several of the discussions, the author has attempted to show by means of a practical problem wherein his theory of designing is best, both in economy of material and subsequent cost, and in ease of application.

Chapter two will contain all the data necessary for designing the three story Reinforced building; the erection drawings and plans of the same will be included in this part; all designs of beams, floor slabs and columns and such other details as would be required by the contractor for the complete erection of the proposed structure will be shown. The application of

the selected formula and the results computed by using these is embodied in Chapter 2. Only such architectural principles have been employed as will aid in the general appearance of the building, and at the same time furnish a structure best fitted for the purpose to which such a building could be put.

In Chapter three will be found the specifications and regulations governing the erection of the proposed building. Such limitations and rules have been incorporated in this section as will bind the contractor to honest work and insure the erection and completion of the building in a thoroughly workman-like manner.

Regulations governing the quality and mix of the concrete, the building of forms, placing of reinforcement and concrete are therein specified.

Chapter I.

The subject of beam theories will be considered first. The present trend of thought on the flexure of beams seems to be to develop a rational formula and, after securing such a one, to determine as closely as possible the value of certain factors contained therein, inserting these approximations, neglecting other unimportant factors, and then reducing and simplifying these rational formulas until they take the form of straight line formulas. A careful study of the various theories has caused me to choose the one presented by Prof. Hatt to the Western Society of Engineers. His formulas when reduced become very simple in application.

The theory of the reinforced concrete pieces, strained in flexure, is usually based upon the following assumptions.

First:- That sections plane before bending remain plane surfaces after bending, within the elastic limit of the steel.

Second:- The applied forces are parallel to each other and perpendicular to the neutral surface of the beam before bending.

Third:- The values of the coefficients of elasticity obtained in direct tension and compression apply to the material under stress in beams.

Fourth:- There is no slipping between the concrete and steel reinforcement, but the two form so perfect a union that they will act together as practically homogeneous material.

Fifth:- There are no initial stresses in either the concrete or the steel due to shrinkage of the concrete while setting.

Sixth:- The entire tensil stress is carried by the steel.

(a) The assumption that sections plane before bending remain plane after bending within the elastic limit of the steel, is universally adopted. Numerous tests have been made to authenticate this theory and the results agree so closely, that the designer is safe in accepting the assumption.

(b) It is essential for the purpose of analysis that the applied forces shall be parallel to each other and perpendicular to the neutral-axis; otherwise it will be necessary to resolve the forces into components parallel and perpendicular to the neutral axis. Such an operation will introduce another unknown quantity and complicate the problem.

(c) It is assumed that the coefficient of elasticity of concrete is variable within the limits of stress and an assumption is always made as to the form of the stress strain curve of concrete in compression. There are various theories advanced for the form of this curve, but in the discussion following I assume that the curve is a parabola.

(d) As has been stated, the utility and safety of a reinforced concrete structure depend largely upon the bond between the two materials. In order to increase this union, mechanical means are employed and should be used so that the stresses may be more easily transmitted from the concrete to the steel. It is desirable in designing reinforced beams to use a number of small bars in preference to one or more large bars of equal area; by so doing the two materials approximate more closely a homogeneous material.

(e) Experiments have shown, and it is universally accepted,

that initial stresses are set up in the steel and concrete due to shrinkage of the concrete while setting, but these stresses are so small that they can be neglected in developing a formula for flexure, without materially endangering the life of the structure.

(f) The results of recent tests seem to indicate that the tensile strength of concrete should be neglected; at best the allowable tension on concrete is very small and we are on the safe side in assuming the tension to be carried entirely by the steel. It is advisable that the tension in the lower part of the beam shall not be so great that the elongation of the concrete shall exceed .001 of its length.

In explanation of the following formulas the symbols used will designate the respective terms as follows:

Let E_s = Modulus of elasticity of steel.

E_c = Modulus of elasticity of concrete in compression.

F_s = unit tensil stress in steel.

F_c = unit compressive stress in concrete.

A_s = Area of steel in tension.

A = Area of cross-section of concrete from the upper face of beam to center of reinforcement. $A = bd$.

λ_s = unit elongation of steel in tension.

λ_c = unit compression of extreme fibres of concrete in compression.

b = breadth of beam in inches.

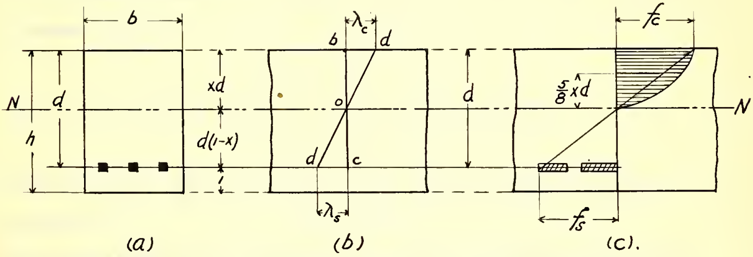
d = depth of reinforcement below compression face of beam or the effective depth.

xd = distance of neutral axis from compression face of beam.

$p = \frac{A_s}{A}$ = ratio of cross-section of steel in tension to area of concrete above center of gravity of the steel.

$e = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to concrete taken at 12.

For simplicity assume a rectangular beam under flexure; the accompanying figures will illustrate the section and give a graphical representation of the deformation and stresses. The above symbols being applied to the figure serve to illustrate more fully the operations. Applying the fundamental principles and assumptions as enumerated, we may deduce the following equations.



In the above diagram N N represent the neutral axis Section (a) represents a cross-section of beam, fig. (b) the deformation diagram and fig (c) the graphical representation of the distribution of stresses. λ_c will be the deformation of the extreme fibre of the concrete in compression. λ_s the unit elongation of the steel.

Now from (1) Since sections plane before bending are

plane after bending it follows that,

$$\frac{\lambda_c}{\lambda_s} = \frac{xd}{d\ (1-x)}$$

By definition $E_c = \frac{Pl}{A\lambda_c} = \frac{fc}{\lambda_c}$ for unit length.

and $E_s = \frac{fs}{\lambda_s}$ "

Therefore $\lambda_c = \frac{fc}{E_c}$ and $\lambda_s = \frac{f_s}{E_s}$

" $\frac{\lambda_c}{\lambda_s} = \frac{\frac{f_c}{E_c}}{\frac{f_s}{E_s}} = \frac{xd}{d\ (1-x)} = \frac{f_c}{E_c} \times \frac{E_s}{f_s}$

Solving for f_c $f_c = \frac{E_c\ x\ f_s}{E_s\ (1-x)}$

Now the total stress on the concrete above the neutral axis is represented by the area within the parabola when the parabolic distribution of stresses is assumed, for a unit width this stress equals

$$2/3\ x\ d\ f_c$$

and for a beam of width b the total compression

$$F_c = 2/3\ x\ d\ b\ f_c$$

Now the total tension in the steel equals, area times unit stress or $F_s = A_s\ f_s$

For equilibrium in the section these two parallel and opposite forces must be equal, otherwise as a couple rotation would result.

Then $A_s\ f_s = 2/3\ x\ d\ b\ f_c$

dividing by bd. $\frac{A_s\ f_s}{bd} = 2/3\ x\ f_c$

but bd = A and by definition $\frac{A_s}{A} = \rho$ (percentage)

Therefore:- $pf_s = 2/3 \; x \; f_c$

but $f_c = \frac{E_c \; x \; f_s}{E_s \; (1-x)}$ substituting in above we obtain

$$pf_s = \frac{2 \; E_c \; x^2 \; f_s}{3 \; E_s \; (1-x)} \quad \text{or } p = \frac{2 \; E_c \; x^2}{3 \; E_s \; (1-x)}$$

Now reducing: $2/3 \; x^2 = \frac{E_s}{E_c} \; (1-x) \; p$

from which $2/3 \; x^2 + \frac{E_s}{E_c} \; p \; x - \frac{E_s}{E_c} \; p = 0$

Substituting $\frac{E_s}{E_c} = e$ Then $2/3 \; x^2 + e \; p \; x - e \; p = 0$

Solving the quadratic for x

$$x = \frac{-pe \pm \sqrt{\overline{pe}^2 + 8/3 \, pe}}{4/3}$$

or

$$x = -\frac{3}{4} pe \pm \sqrt{\frac{3}{2} pe \left(1 + \frac{3}{8} pe\right)}$$

As in practice the values of e and p are assumed and by solving this equation x may be found. Finally, when the position of the neutral axis is known, the moment of resistance of the beam may be found. The center of moments being located at the neutral axis, the total resisting moment of the beam is equal to the sum of the moments of compression in the concrete and tension in the steel. Let M_r represent the resisting moment of the beam.

Then $M_r = 2/3 \; x \; d \; f_c \; b. \; 5/8xd + A_s \; fs \; ^{d} \; (1-x)$

or

$M_r = \frac{5}{12} \; f_c \; b \; x \; d^2 + As \; fs \; d \; (1-x)$

Now in order that the beam shall be safe the resisting moment must be equal to or greater than the bending moment.

$$\text{or } M_r = M.$$

$$\therefore M = \left(\frac{5}{12} f_c x^2 + \frac{A_s}{A} f_s (1-x)\right) bd^2$$

Since $bd = A$ and $\frac{A_s}{A} = p.$

$$M = \left\{5/12 f_c x^2 + p f_s (1-x)\right\} bd^2 \qquad (a)$$

The coefficient in this equation contains both f_c and f_s which is rather an inconvenient form, as it is usual, when a definite value of $\frac{A_s}{A}$ or p has been determined for use in the calculations, to assume a safe working value for either f_c or f_s, compute the section of the beam and then determine if the working value of f_s or f_c as the case may be, for this section comes within safe limits.

We still reducethe equation for M as follows:

From equation: $pf_s = 2/3\, x\, f_c$

$$f_s = \frac{2\, x\, f_c}{3\, p} \quad \text{and } f_c = \frac{3pf_s}{2\, x}$$

Substituting these values successively in equation

(a) $M = pf_s \left(1 - \frac{3}{8}\, x\right) bd^2$

to be used when the allowable unit stress in the steel is assumed or

$$M = \frac{2}{3}\, x\, f_c \left(1 - 3/8\, x\right) bd^2$$

when the allowable stress in the concrete is assumed.

It can be readily seen that the coefficient $pf_s \left(1-\frac{3}{8}x\right)$ will be the governing factor when a low percentage of steel is used, or when concreteof high strength is assumed. And the

coefficient $2/3xf_c\ (1-\frac{3}{8}x)$ will be the determining factor when a high percentage of steel is used, when a steel with high elastic limit is used or when a concrete of low crushing strength is used.

If definite values for p, e, f_c and f_s be assumed then these two coefficients result in constant quantities and may be represented by C. Then c may be either $2/3\ x\ f_c\ (1-\frac{3}{8}\ x)$ or $p\ f_s\ (1-\frac{3}{8}\ x)$.

Then the equations necessary for the solution of a problem in beam designing are

$$x = -\frac{3}{4}\ e\ p + \sqrt{\frac{3}{2}ep\left(1+\frac{5}{8}ep\right)}$$

$$2/3\ f_c\ x = p\ f_s$$

and $$M = C\ bd^2$$

By assuming values for p, e, f_s and f_c two values of C will be obtained and the smaller of the two should be used for computing the resisting moment of the beam.

The foregoing theory and equations based on legitimate assumptions, though based on a parabolic distribution of the stresses which some designers object to appears to the author to fulfill all the requirements and result in members, both economical in material and cost. At the same time these formulas give sizes to members which correspond very closely to results computed by experience and what is more they stand test.

The above formulas will be used in the designing of all beams and girders; the floor slabs will be considered as simple beams and so designed as to have their length from center of supports equal to the distance between the outside

of beams. The reinforcing in the slabs running parallel with the floor beams, will be of such amount as will safely provide for temperature stresses in the slab and prevent cracking. Results from actual practice will be consulted for determining the required amount.

The beam formulas developed by Mr. Edwin Thatcher M-Am-Spc. C. E. deserve careful consideration. Though he has made rather irrational assumptions for the values of E_c & E_s, one can after deriving the formulas make whatever assumptions he wishes and reducing the equations his formulas are brought in very simple form and are very easy to apply.

He bases his calculations on the assumption

1st- That sections plane before bending remain plane after bending.

2nd- That the steel and concrete form a homogeneous mass and act together in distribution of the stress.

3rd- That the stress - strain curve is a straight line, or the stresses vary from the neutral axis to the extreme fibre as the ordinates to a straight line. This distribution is based upon the assumption that the modulus of elasticity of the concrete is constant throughout the whole range of stress.

4th- Tensile stress in the concrete is neglected.

5th- The loading is perpendicular to the neutral axis before bending.

Mr. Thatcher has based his formulas on the ultimate safe loads. When applied for designing a factor of safety must be assumed and the results calculated accordingly.

The following symbols will be used in the explanation of these formulas.

F_c = crushing strength of the concrete lbs. per sq. in.

f_c = Compressive strength lbs per sq. in. in concrete

F_s = Tensile strength per sq. in. steel

f_s = Tensile stress in pounds per sq. in. on steel.

E_c = Modulus of elasticity of concrete

E_s = " " " " steel

$e = \frac{E_s}{E_c} = 12.$

A = area of steel in tension for 1 inch in width of beam.

l. = length of beam in feet = span.

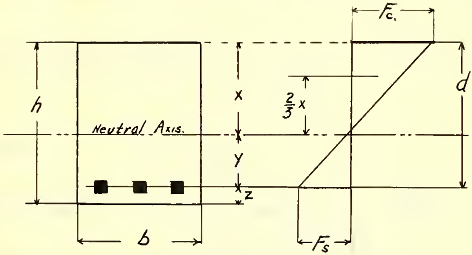
M = Bending moment in foot lbs for 1" width of beam (ultimate

W = load at center including weight of beam for 1 inch in width.

w = uniform load per linear foot including weight of beam for 1 inch in width.

12 w = w' uniform load per sq. ft. (ultimate)

The various distances on the beam are shown in the fig.



As in Prof. Hatts formulas we will assume that there is steel in the tension side only of the beams. From the assumption that sections plane before bending are plane after we have,

$$\frac{x}{F_c} = \frac{y}{F_s} \quad \text{and} \quad \frac{x}{\frac{F_c}{E_c}} = \frac{y}{\frac{F_s}{E_s}}$$

$$\frac{E_s}{E_c} = e \qquad \frac{x}{y} = \frac{\frac{F_c}{E_c}}{\frac{F_s}{E_s}} = \frac{F_c}{F_s}\, e \qquad x = y \cdot \frac{F_c}{F_s}\, e$$

and $$F_c = \frac{F_s\, x}{e\, y}$$

Now $$d = x + y = y\, e\, \frac{F_c}{F_s} + y$$

and therefore,

$$y = \frac{d}{\left(e\, \frac{F_c}{F_s} + 1\right)}$$

also $$x = d - y$$

The conditions of equilibrium require that the total tension = the total compression. Then, since the area $F_c\, x/2$ represents the total compression. $F_c \frac{x}{2} = A\, F_s$ or $x = \frac{2\, A\, F_s}{F_c}$

Substituting in this equation the value of F_c as found

$$x^2 = 2\, A\, e\, y$$

again $$y = \frac{x^2}{2\, a\, e}$$

Substituting value of x from equation $x = \frac{2AF_s}{F_c}$

$$y = \left\{\left(\frac{F_s}{F_c}\right)^2 \left(\frac{2}{e}\right)\right\} A$$

$$d = x + y = \left\{\left(\frac{F_s}{F_c}\right)^2 \left(\frac{2}{e}\right)\right\} A + \frac{2\, A\, F_s}{F_c}$$

$$d = 2\,A\left\{\frac{F_s}{F_c} + \frac{1}{e}\left(\frac{F_s}{F_c}\right)^2\right\}$$

Solving for A $\quad A = \dfrac{d}{2\left\{\dfrac{F_s}{F_c} \quad \dfrac{1}{e}\left(\dfrac{F_s}{F_c}\right)^2\right\}}$

also $\quad d = x + y = x + \dfrac{x^2}{2\,A\,e}$

Therefore $\quad x^2 + 2\,Ae\,x = 2\,Aed$

$$x^2 + 2\,Aex - 2\,Aed = 0$$

$$\therefore \quad x = \sqrt{2Aed + (Ae)^2} - Ae \qquad (1)$$

Now if M = resisting moment of the beam

Mc = " " " concrete in compression

M_s = " " " steel in tension

Taking the neutral axis as the center of moments. Then if x and y be measured in inches

$$M_c = \frac{F_c\ x}{2} \cdot 2/3\ x = \frac{F_c\ x^2}{3} \text{ inch lbs.}$$

Dividing by 12 to reduce to foot lbs.

$$M_c = \frac{F_c\ x^2}{36} \text{ ft. lbs.}$$

and $\quad M_s = F_s\ A\ y$ inch lbs.

or $\quad M_s = \dfrac{F_s\ A\ y}{12}$ ft lbs.

The total resisting moment $= M = M_c + M_s$

or

$$M = \frac{F_c\ x^2}{36} + \frac{F_s\ A\ y}{12}$$

Substituting for Fc its value $\dfrac{F_s x}{e\ y}$

$$M = \frac{F_s\ x^3}{36ey} + \frac{F_s\ Ay}{12}$$

$$= \frac{F_s}{36}\left(\frac{x^3}{ey} + 3\ A\ y\right) \qquad (2)$$

Now for a simple beam uniformily loaded

$$M = \frac{wl^2}{8} \qquad w = \frac{8M}{l^2}$$

$$\therefore\ w = \frac{F_s}{4.5\ l^2}\left(\frac{x^3}{e\ y} + 3\ A\ y\right) \qquad (3)$$

and $$w' = 12\ w = \frac{12\ F_s}{4.5\ l^2}\left(\frac{x^3}{e\ y} + 3\ A\ y\right)$$

For a simple beam loaded at the center

$$M = \frac{Wl}{4}$$

$$W = \frac{F_s}{9\ l}\left(\frac{x^3}{e\ y} + 3\ A\ y\right) \qquad (4)$$

Equations (1), (2), (3) and (4) will furnish all the necessary data for the designing of a beam after values for F_s, E_c, E_s, A, d and l are assumed. The calculations are made with these assumptions and the value of F_c found, if this value exceeds the determined strength of the concrete in compression. The values of M, W or w, as determined by the equations 2, 3 and 4 should be reduced in the ratio that the probable ultimate compressive strength of the concrete bears to F_c.

Mr. Thatcher has made the assumptions for values of E_s, E_c, F_s w etc. and compiled tables, which aid very material-

ly in reducing the calculations. By applying the assumed values in the formulas and reducing the equations, formulas are obtained for x, M, A,W and w which are very simple and can be applied to any case depending on the age of concrete and mixture, or in other words the factor of safety used.

These formulas are, as I have stated, simple in application and are rational; but as compared with Prof. Hatts reductions they are no simpler nor are they based on more rational assumptions; on the contrary they include a greater number of assumptions and though based on the safer distribution of stresses as straight line formulas, their results have given no better satisfaction, but a greater cost.

-- COLUMN FORMULAS: THEORY --

The round hooped columns will be used in the building proposed, therefore, the theory of the spiral hooped column will be discussed and a formula for determining the amount of steel and size of column deduced. Engineers have for some years designed members subject to direct compression, with little knowledge of the action of the members under the load and those that have stood are recorded as safe. Such irrational assumptions may be correct but to design a member from a rational formula based on theory is rather a difficult problem. However, I have tried to deduce, or compile I may say a formula, based as nearly as possible on the true theory of members subject to direct compression.

In recent years numerous tests have been conducted

on miniature columns but very few experiments have been performed on members properly proportioned, and the designer is forced to accept the results of these tests and make his calculations accordingly.

These tests have been very conclusive in proving that a compression member does not fail from shearing along planes inclined to the axis but that the concrete seems to break in longitudinal planes or parallel to the axis. From such tests one is lead to believing that longitudinal reinforcement in the columns adds little to the strength of the member. There esist such uncertain theories as to the action of columns with straight reinforcement, that it is desirable to use comparatively low working stresses in the concrete. Factors of safety from 6 to 10 are none too large.

If the designer can distribute the reinforcing metal so that the column will not fail as explained above, then the life and economy of the structure are assured.

The resisting power of concrete may be increased by reinforcing it against lateral yielding either by shearing in a vertical or diagonal direction or by preventing the concrete from spreading laterally as shortening takes place. From tests on concrete pieces it has been found, that when a block of concrete is subjected to heavy pressure the cohesion between the molecules is lessened as the block decreases in height and increases in size in a direction perpendicular to the line of pressure. This tendency of the molecules to flow horizontally is resisted by the cohesion and friction between

the molecules. Now we see that if we can confine the pillar of concrete from this spreading, by the use of spiral reinforcement or by surrounding it with a tube of metal, the resistance to compression of the concrete will be greatly increased.

The maximum degree of efficiency will be reached when the hooping is continuous and of sufficient strength and rigidity to retain the component parts of the concrete within definite limits. From tests on concrete in compression it has been found that the leaner mixtures are most compressible. So in hooped concrete the rich mixture should be employed, since it is desired to have only a minimum compression on the concrete before the hooping is brought into action.

Some designers employ the longitudinal reinforcing rods in the columns for secondary stresses only, claiming that previous to the removal of the forms, preventing the spreading of the concrete the rods will carry a great part of the load and prevent rupture in the concrete subsequent to final hardening. Such an assumption is surely safe, but at best is, in a measure a waste of steel and concrete, because the sections of columns must be materially increased to carry the final load, when according to their assumptions the steel is not stressed.

There is no question, but that the longitudinal rods carry part of the direct compression and assist the concrete as long as they are not stressed beyond the elastic limit. When such conditions exist the longitudinal rods are purely a detriment to the column, for they will surely buckle and tend to separate the layers of concrete in vertical planes.

Considére has given a formula determining the strength of columns reinforced both longitudinally and spirally as follows. Let $A_s 1$ = the sectional area of an imaginary longitudinal reinforcement equivalent to the hooping. Then if A_s = area of the rods and A_c = area of concrete, $e = \frac{E_s}{E_c}$

F_{sc} = elastic limit of metal in compression and

F_{ss} = elastic limit of hopping in tension and

F_c = ultimate strength of concrete in compression

$$P = F_c A_c + F_{sc} A_s + 2.4 F_{ss} A_s l$$

or $P = F_c (A_c + e A_s + 2.4 e A_s l)$

The constant 2.4 is the ratio of the resistance of the concrete due to the hooping to the resistance of the concrete due to longitudinal reinforcement. This constant has been determined by experiment on sand confined in tubes. In the above formula the calculations are based on ultimate strengths of both concrete and steel, a factor of safety must therefore be introduced.

The above formula is very ambiguous and omits any reference to the pitch of the hooping.

A more rational formula based upon experiments performed by Considére will be given, however, in this formula. No account is taken of longitudinal reinforcement.

In his experiments he found that the compressive resistance pf a prism of mortar or concrete per unit of area equals $1.5 f_c + 4.8 p'$, when f_c is the natural unit compressive resistance of the concrete, and p is the unit pressure exerted by the hooping on the whole of its lateral surface.

Now let R = pressure per sq. in. on prism,

d = diameter of prism and hooping,

p = external radial unit pressure,

z = distance between adjacent coils of hooping, for single coils equal pitch,

p' = uniform unit pressure exerted by the hooping,

f_c = unit compressive stress of concrete,

f_s = " tensile stress of hooping,

A = sectional area of column,

A_s = sectional area of one coil hooping metal.

R_c = unit compressive resistance of concrete due to hooping = $1.5\ f_c + 4.8\ p'$.

Considére develops this formula by reference to the principles of hydrostatics, we are then privileged to say that, the internal pressure on the concrete will be resisted by an equivalent external pressure caused by the hooping.

The force tending to cause horizontal rupture is $R\ d\ z$.

The tensile stress of the hooping which resists this force is $2\ A_s\ f_s$

Therefore $2\ A_s\ f_s = R\ d\ z$

For the conditions of equilibrium R must be resisted by an external pressure p.

$$\text{or } 2\ A_s\ fs = p\ d\ z$$

$$\text{whence} \quad p = \frac{2\ A_s\ f_s}{d\ z}$$

Substituting p in $1.5\ f_c + 4.8\ p$.

$$R_c = 1.5\ f_c + \frac{9.6\ A_s\ f_s}{d\ z}$$

Then the total strength P of the column will be

$$P = A\ R_c = A\ (\ 1.5\ f_c + \frac{9.6\ A_s\ f_s}{d\ z}\ )$$

$A = 1/4 \pi d^2$ $\qquad P = 1.2 f_c d^2 + \frac{7.5 A_s f_s d}{z.}$ for circular columns.

This formula can be readily applied when values for f_c f_s and d have been assumed. The value of As can be calculated and the necessary amount of metal in hooping determined for the successive loadings.

There have been but very few experiments performed on concrete columns having both longitudinal and horizontal reinforcement. Considére, the French Engineer, has given this subject some consideration but I am not at all satisfied with the formulas I am obliged to use. A large field in testing awaits some progressive Engineer and more may be learned regarding the action of such members.

If columns are so designed that their least width is not less than one twelfth of their length the failure of such a member due to flexure need not be taken into account. Any such action will be easily carried by the vertical rods bonded to the concrete.

-: Design of floor Slabs:-

As in the design of beams, so is it with the slab; considerable difference of opinion exists as to the manner in which they should be proportioned and the metal required as reinforcing. It seems and proves very satisfactory to design these slabs as simple beams uniformily loaded; placing enough steel in the lower portion of the slab to provide for tension, and enough steel transversely with the member to care for temperature and initial setting stressos.

The method employed in the concrete building will be as here

outlined. The bending moment being taken as $\frac{Wl}{10}$ or $\frac{wl^2}{10}$; there is some discussion among Engineers as to the correct value of M, but the above value is certainly conservative, and when one considers that a slab is continuous over a least one support this value is not a reckless guess. At present American engineers are designing slabs with the above formula and obtain sections amply able to carry the loads.

Cross reinforcement that is, steel rods parallel to the principle beams upon which the slab rests in addition to the principal bearing rods, as stated, is customarily used to prevent shrinkage and temperature cracks and necessarily gives added strength. If expansion joints are provided at frequent intervals the necessity of these rods is questioned, but most generally the expansion joint is unsightly and affords an excellent place for dirt to collect.

Mr. A. L. Johnson has attempted a mathematical demonstration of how to prevent cracking due to temperature. He states that the quantity of metal used should be enough to equal the tensile strength of the concrete at the elastic limit of the metal. Thus if we call the tensile strength of a 1 - 2 - 4 mixture 200 lbs per sq. inch, and the elastic limit of mild steel 36,000 pounds per sq. inch, the number of sq inches of steel required would be $\frac{200}{36000}$ or $\frac{1}{120}$ of the number of square inches in the slab section. Such amount must be distributed uniformily throughout the member.

- SHEAR -

The subject of shear and diagonal tension in beams has been and is one of considerable importance; fully as many beams under

test have failed from these causes as have failed by compression, or direct tension. The value of concrete in shear depends on the quality of the concrete and the manner in which it has been placed. It is true the maximum shear in a beam occurs at the supports and is equal to the greatest reaction; it seems within the limits of conservative design to assume the concrete taking this shear at a certain value and placing enough steel within the section determined by the size of beam to carry the excess shear. The matter of determining the shear at points along the beam and providing for such stress when the concrete is unable to care for it is one of considerable importance.

From mechanics of beams we know that there exist throughout a beam vertical and horizontal shearing stresses of varying intensities, and that at any point in the beam the vertical shearing unit - stress is equal to the horizontal shearing unit-stress there developed. The horizontal shearing stresses tend to disrupt the bond between the steel and concrete, and the addition of a mechanical bond, such as deformed bars, greatly increases the allowable shear. Very few beams have failed, due to horizontal shear alone for these stresses are comparatively small, but the vertical shearing stress is of considerable value. In order to determine the shear at all points of the beams and girders of the machine shop building; the shear diagrams for the particular loadings will be constructed. From these diagrams one can readily measure the shear, both vertical and horizontal, and test the beam against failure due to either cause.

In many test beams, failure has been recorded, due to diagonal cracks running from the under side of the beam to the neutral axis. These diagonal cracks were formerly attributed to the failure of the concrete in shear, but recent tests indicate that the cracks, at least in part, are due to internal tension caused by the stretching and slipping of the rods employed in the reinforcement.

To prevent these diagonal cracks, the steel should be so designed and placed as to give the greatest possible adhesion to the concrete, and thus prevent slipping as the steel becomes reduced by the stretch. An extra precaution against shear and tension is obtained by placing inclined or vertical rods at intervals in the beam either as separate stirrups or unit frames.

Theoretically, the slope of the reinforcing rods should be 45^0, inclining away from the center of the beam, but because of the difficulty in placing and retaining rods in this position, they are more frequently placed vertical.

There have been various methods suggested by engineers for placing of these U - bars or stirrups. Some are purely empirical while others are based upon mathematical deductions.

Mr. E. L. Ransome's empirical rule for placing the stirrups is to place the first a distance from the end of the beam corresponding to one quarter the depth of the beam, the second a distance of one half the depth of the beam beyond the first, the third a distance of three-quarters the depth of the beam beyond the second, and the fourth a distance of the depth of beam beyond the third, continuing at this spacing as

far as the stirrups are required. Others make it a rule to space them the same distance apart as the depth of the beam, or slightly closer than this so that any diagonal line at an angle of 45° with the neutral axis will pass through the stirrups.

The Prussian requirements state that the stress must be calculated as actual shear and rods enough placed in the concrete so that this shear shall not exceed 65# per sq. inch.

A formula derived by Mr. J. W. Schaub and given in the Engineering News, April 16, 1903, p. 348 is as follows:

Let s = Area of steel required in stirrup at any section of beam sq. inches.

A = total sectional area of beam in sq. inches.

p = ratio of cross-section of horizontal steel to cross-section of beam.

pA = Area of horizontal steel in sq. inches.

l = length of beam in feet.

z = distance from end of beam to section where stirrup is required, in feet.

Then

$$s = \frac{4pA}{l}\left(1 - \frac{2z+1}{l}\right)$$

This formula gives very satisfactory results compared with practice and having been derived from purely theoretical assumptions is one worthy of notice. In the above deductions Mr. Schaub has considered the beams to be uniformly loaded.

Chapter II.

Design and drawings of Three Story and basement reinforced concrete machine shop building.

80' x 120'.

Data:-

The proposed building is to be constructed with a reinforced concrete skeleton, and vitrified brick curtain walls.

Span of floor beams 20' - 0" c to c. Span of Girders 16' - 0" C to C. Span of slabs 1st 2nd and 3rd floors 5' - 4" center to center.

Span of roof slabs 8' - 0" C to C.

Live Loads Roof = 75# per sq. ft.
3rd floor =275 # per sq. ft.
2nd " =300 # " " "
1st " =300 # " " "
Bsm't." =250 # " " "

Dead loads:-

Composition Roofing 7# per sq. ft.
Concrete(Stone) 150# " cu. ft.

Corrugated Bars.

Size	Net Section Sq. in.	Weight in lbs. per ft.
3/4"	0.56	1.91
7/8"	0.77	2.60
1 "	1.00	3.40
1¼"	1.56	5.31

Twisted Bars.

Size	Net Section Sq. in.	Weight in lbs. per ft.
$\frac{1}{4}$"	.062	.213
3/8"	.14	.478
$\frac{1}{2}$"	.25	.850
5/8"	.39	1.328
3/4"	.56	1.913
7/8"	.765	2.60

Unit Stresses.

1-2-4 Unit Compressive Stress in Concrete f_c = 700 lbs.per sq.in.

1-3-6 (Foundations) " " " f_c = 500 " " " "

1-2-4 (Columns hooped) " " f_c = 750 " " " "

1-2-4 Shear unit stress" " = 65 " " " "

Unit shearing stress in steel = 10000" " " "

f_{sc} " Compressive " in steel = 12000" " " "

f_s " Tensile " in steel = 14000" " " "

E_s Modulus of elasticity of steel =30,000,000

$\frac{E_s}{E_c}$ Ratio of Moduli of elasticity of steel and concrete to be taken from Chicago building Code = 12

E_c = 2,500,000

		C	C
p	.x	f_s =14000	f_c= 700
.01	.319	123	131
.015	.412	178	163
.02	.463	232	178
.025	.505	284	194
.03	.543	336	202
.04	.607	437	217
.05	.660	532	235

$M = \frac{wl}{10}$ or $\frac{wl^2}{10}$ for beams

$M = \frac{wl}{10}$ or $\frac{wl^2}{10}$ for slabs

L = Clear span in all cases.

$M = \frac{wl}{8}$ Roof Girders. Concentrated loading.

DESIGN OF MEMBERS.

Roof Slab:-

We will assume a 4" slab reinforced with 1% steel.

Dead Load: = Wt. of Concrete 50# sq.ft.
" " Roofing 7# " "
Live Load = 75# " "
Total 132#

$M = \frac{wl^2}{10}$ L = 7'-0" clear span.

$$M = \frac{12 \times 130 \times 49}{10} = 7650 \text{ inch lbs.}$$

P = .01 X = .319 C = 123. $M = C\,b\,d^2$
Take slab 1 ft. wide

$$d^2 = \frac{7650}{123 \times 12} = 5.2$$

$$d = 2.7''$$

Therefore 3½" slab is ample

.01 steel = .42 sq. inches

Use 2 - 3/8" twisted rods 6" c to c

Temperature reinforcement = $\frac{1}{120}$ of longitudinal.

Use 2 - $\frac{1}{4}$" twisted bars 6" c to c.

ROOF BEAMS: R,

D.L. from roof = 130 # sq. ft.

Panel size = 8 x 16

Total D.L. from roof = 16,650 #

D.L. of beam assuming 9" x 9" Total depth 13"

= 1540 #.

Total weight W = 18,190 #

$$M = \frac{wl}{10} = \frac{12 \times 18190 \times 19}{10} = 414{,}700\text{" lbs.}$$

$$414{,}700 = 178\ bd^2$$

Assuming 2 % Steel C = 178.

$$bd^2 = 2330$$

$$b = 10" \quad d = 15"$$

2% Steel = 3 sq. in. Use 3 - 1" Corrugated bars.

$$\text{Shear at end} = \frac{18190}{2} = 9090\ \#$$

150 sq. in. concrete @ 65 # = 9750 #.

Diagonal Tension.

3/8" U-bars spaced 6" - 6" - 8" - 12" - 15"

ROOF GIRDERS:

D.L. from roof = 4x12x130 = 6240 #

D.L. of Girder)
) = 150 x 14.5 = 2175
Assuming 10x14)

Total 8415 lbs.

Concentrated load from beams = 18 000 #

$M = \frac{Pl}{8}$ and $\frac{wl}{10}$

M = 537,900 in.lbs. Assuming p = .03

C = 202 and b = 10"

$$M = 202 \times 10 \times d^2$$

$$d = 15"$$

3% Steel = 4.5 sq. in. Use 3 - 1¼" bars.

Maximum shear = 13210 #

" " allowable in Concrete = 9750 #

Shear to be carried by steel = 3460 #

This amount will be provided for by two tie bars 3/4" dia. running through the column.

Tie bars 3/4" x 4' long, wired at joint.

Stirrups. Use 3/8" U-bars spaced same as in roof beams.

For negative Bending place 2-3/4 x 6' bars over column.

3rd STORY COLUMNS.

D.L. from roof direct @ 130# sq. ft.	=	22,825#
D.L. " 2 roof beams,	=	4,350 #
D.L. " Girders,	=	1,725 #
Total load	=	28,900#

From Considére formula $P = f(A_c + eA_s + 2.4\,eA_s l)$

Assuming the column hooped with ¼" spiral hooping 2½" Pitch $A_s = .05$ sq. in.

dia. of 10" $A_c = 78.5$ sq. in.

Allowable P=750 (78.5 + 12x.05)

" P = 60,000 #

It will be seen that in these columns longitudinal rods are not required. The concrete alone will more than carry the load. From the mechanics of columns, it is not good design to have a column diameter less than $\frac{1}{12}$ the length. Therefore a 12" round column will be used.

The wall columns for 3rd floor will be taken at 8" x 14". This size being more than is actually required for the load but to give rigidity and strength against excessive wind loads the assumption is not irrational. Reinforce with 4-$\frac{1}{2}$" Twisted Rods tied every 12" with 1/8" iron wire.

3rd FLOOR SLAB:

Assume a 4" slab reinforced with 1% steel.

$p = .01 \quad C = 123. \quad l = 5'\text{-}0".$

Weight of slab one foot wide x 5' = 250 #	250#
Load from floor assuming that never more than 90% is actually applied. = .90 x 275 = 247# x 5' =	1235#
	Total load = 1485#

$$M = \frac{wl}{10} = \frac{1485 \times 5 \times 12}{10} = 8910'' \text{lbs§}$$

$$\therefore \quad 8910 = 123 \times 12 \times d^2$$

$$d = 2.5''.$$

A 4" slab is much too large, but for factory purposes a slab less than 3" is frail.

Use a 3½" slab reinforced with 3/8" twisted bars

spaced 6" c to c.

Temperature rods 3/8" twisted 12" c to c.

3rd FLOOR BEAMS:

LL. = 16 x 5.33 x 275 x .90 = 20,100#

Assuming not more than
90% of total load applied
at one time.

85 sq. ft. slab at 45# = 3,625#

Assumed weight of beam = 1,200#

Total = 24,925#

$$M = \frac{wl}{10} = 569{,}200\text{ " lbs.}$$

Assuming 3% steel in the beam. p = .03 c = 202.

$$569{,}200 = 202 \times 10 \times d^2$$

$$d = 16\tfrac{1}{2}" \quad b = 10"$$

3% steel=5 sq. in.

Use 3 - 1 1/4" corrugated bars.

Shear = 12,460# (See diagram 3 - A.)

Concrete at 65# per sq. in. = 10,725# allowable

For safety and negative bending place 2 - 7/8" corrugated bars 9'-0" long over girder.

Use 3/8" twisted bars for stirrups spaced 4" - 8" - 10" - 12" - 16" - 16".

3rd FLOOR GIRDERS:

l = 15'-0" p= .02 c= 178.

Load from 2 beams, assuming 90% of total = 49,850#

D. L. on Girder direct = 2,160#

Assuming a Girder 14" x 18" weight = 3,500#

Total load = 55,510#

$$M = \frac{55,510 \times 15 \times 12}{10} = 991,800 \text{ in. lbs.}$$

$991,800 = 202 \times 14 \times d^2$ d= 20" b= 14"

A= 280 sq. in. A_s= 5.6 sq. in.

Use 4 -1 1/4" corrugated bars.

Max. Shear = 27,750#. Placing a 1 ft. bracket at 45°

A= 448 sq. in. Concrete in shear @ 65# Allowable shear 28,900#. To reinforce against negative bending and to tie consecutive girders. Place 2-7/8" corrugated bars. 8" c to c over column in center of slab. Tie lower reinforcement of girders with 2 - 7/8" bars wired at joints. Brackets 1' x 1' reinforced with 2 - 3/4" bars. 6" c to c.

Shear at point 2' from end= 24,100# (See diagram 3-B)

Allowable shear in concrete at this point = 18,200#

The bars in lower reinforcement and over column will provide for any excess in this design.

For stirrups use 3/8" U-bars, spaced according to Ransome's rule.

5"-10" - 10" 10" - 10" - 15"

2nd STORY COLUMNS:

Load from roof columns. = 28,900#

Load coming directly into column	= 2,720#
Weight from 1 Girder load	=55,510#
" of 3rd Story Column	= 1,540
" from 1 Beam directly	=27,900#
Total load	= 116,570#

$P = f_c\ (A_c + eA_s + 2.4\ e\ A_s \ell)$ Assuming a 14" round column. 1/4" spiral reinforcement of 2 1/2" pitch.

$A_c = 154$ sq." $A_s = .05$ sq. "

Solving for $A_s \ell$ it is found vertical rods are not necessary, but to insure against flexure and for wind bracing use 4 - 1/2" twisted rods inside of hooping. Then 2nd story columns will be 14" dia. 1/4" spiral and 4 1/2" bars for reinforcement. When tested with Considere's other formula the above values are correct.

2nd and 1st FLOOR SLAB:

L. L. at 300# per sq. ft.

Girder span = 15 feet. beam span = 19 feet.

Slab span = 5 feet.

Assume 4" slab. $p = .01$ $c = 123.$

Weight of slab 1' wide	= 250#
Load from floor @ 90% of total	=1350#
Total load	=1600#

$M = 9600$ " lbs.

$$9600 = 123 \times 12 \times d^2$$

$$d = 3"$$

Use 4" slab reinforced with 3/8" twisted bars spaced 6" c to c.

Temperature reinforcement 3/8" twisted rods, spaced

12" c to c.

1st & 2nd FLOOR BEAMS:

Load from floor @ 300# per sq. ft. 5.3 x 300 x 16

Assuming 90% maximum = 22,360#

Weight of slab @ 50# per sq. ft. = 4,250#

Assume beam 10" x 12" Weight = 2,375#

Total 28,985#

Use 2% of steel then c = 178

M = 660,850 in. lbs.

Using 11" breadth d = 18 1/2"

A_c =203 sq. in. A_s = 4 sq. in.

Use 4 - 1" corrugated bars.

Max. Shear = 14,490#. Concrete @ 65#

A_c = 203 x 65 = 13,195#

Place 2 - 3/4" bars. over Girder to provide for excess shear and negative bending.

Shear diagram 1 - 2 - A gives maximum shears for points along beam, at 2 feet from support V = 12,800#. It will be seen that the concrete alone will carry this shear and as the rods placed over the girder extend beyond this point there is absolutely no chance for failure due to shearing stresses.

Place U-bars 9" - 12" - 12" - 15" - etc. c to c

1st - 2nd FLOOR GIRDERS:

l = 15' Assume p = .02 c = 178

Uniform loading from slab direct = 12,950

Weight of Girder 18"x 21" = 5,850

Total uniform load 18,800

Concentrated load = 2 loads of 28,985# each acting at 5' from end of girder.

$$M = \frac{18800 \times 15 \times 12}{10} = 1,739,100 \text{ in. lbs.}$$

$$\begin{aligned} M &= 28,985 \times 5 \times 12 = 338,400 \text{ " } \quad \text{"} \\ \text{Total} &= \underline{}2,077,500 \text{ " } \quad \text{"} \end{aligned}$$

Assuming $b = 18"$ $d = 26"$

$A_c = 468$ sq. in. $A_s = 9.4$ sq. in.

Use 6 - 1 1/4 " corrugated bars.

Shear end of Girder = 38,380# (See diagram 2 - B)

Placing a 1' bracket at 45° under girder.

$A_c = 648$ sq. in. @ 65# = 42,920 lbs.

To provide against negative bending place 3 - 7/8" bars over top of girder.

Shear at point 2' from end = 36,000#

$A_c = 468$ sq. in. Concrete @ 65# = 30420# allowable

It will be necessary to extend the 7/8" bars into the girder past the point, make rods 9 feet long.

Stirrups use 1/2" U - bars spaced

7" - 14" - 18" - 18" - 18" - 24" etc.

1st STORY COLUMNS:

Load from 2nd Story Column = 116,570#

" going into column directly = 3,240#

" from main girders 2 x 38,380# = 76,760#

" " *floor beam direct* = *28,980#*

Weight of 2nd and 3rd Story Columns = 4,300#

Total column load = 229,850#

$$P = K\left(1.2 f_c d^2 + \frac{7.5 A_s f_s d}{z} \right)$$

Assuming A_s as 1/4" spiral reinforcement

A_s = .05 sq. in.   d = 18"   For a 1 - 2 - 4 concrete

K = .7   Pitch of spiral = 2.5"

Then P by test = 230,300 lbs. which is greater than the maximum load.

Reinforcement to consist of 1/4" spiral 2.5" pitch and 6 1/2" twisted rods.   d = 18"

B'SMT COLUMNS:

Load from 1st Story Columns = 229,850#

" entering column directly = 3,240#

" " " from main Girders 76,760#

" " " " floor beams = 29,000#

Weight of a 22" column 390# per ft. = 4,700#

Total column load = 343,550#

$$P = k\left(1.2\, f_c\, d^2 + \frac{7.5\, A_s\, f_s\, d}{z} \right) \qquad k = .7$$

Assume a 22" column, reinforced with 5/16" spiral rods

1-2-4 concrete   Pitch of hooping = 2.5"   Testing,

Allowable load = 309,200#

A_s = .07 sq. in.   Make dia. 23"

Place 6 3/4" twisted rods within hooping.

2nd STORY WALL COLUMNS:

Load from cornice and lintels = 9,600#

Weight of 3rd Story Column = 1,250#

Load from floor and Girders = 58,250#

Total column load = 68,100#

$$P = f_c\, A_c + f_c\, e\, A_s$$

Use 4-1/2" twisted bars. $A_s = 1$ sq. in.

$A_c = 80$ sq. in. Make column 14" x 10"

$b = 14"$ $d = 10"$

Tie rods every 12" with 1/8" wire.

1st STORY WALL COLUMNS:

Load from 2nd Story Columns = 68,100#

Weight of " " " = 2,100#

Load from 2nd floor slab and Girders = 38,380#

" " lintels = 14,000#

Total column load = 122,580#

$$P = f_c A_c + e f_c A_s$$

A_c necessary assuming 4 - 3/4" twisted rods. $A_s = 2.2$ sq. in. $A_c = 137$ sq. in.

Make column 14" x 15" Tie rods every 12" with 1/8" wire

BS'MT WALL COLUMNS:

Load from 1st story columns = 122,580#

Weight of " " " = 2,300#

Load from 1st floor slab & Girders = 38,380#

" " lintels and walls = 14,000#

Total column load = 177,260#

Assuming reinforcing rods 3/4" tied at 12" intervals.

A_s for 4 rods = 2.2 sq. in.

A_c necessary to carry load = 210 sq. in.

Make columns 18" x 14"

4 rods 3/4" twisted.

Total foundation load from wall columns = 182,300#

Maximum foundation load from interior columns=348,600#

Max. load upon wall footing. = 231 00 # per ft. long.

FOUNDATIONS FOOTINGS. Interior Columns.

Allowable pressure per sq. ft. on dry sand as given by Chicago building Code = 4000#. Therefore area of footing required = 88 sq. ft. Make footings 9 1/2' x 9 1/2' Reinforced with 1/2" twisted rods placed both longitudinally and transversely @ 6" c to c.

WALL COLUMN FOOTINGS 7' x 7' reinforced with 1/2" rods @ 6" intervals. Same as in interior footings. Assuming 18" concrete walls, footing area for 1 ft. of wall 7 ft high = 6 sq. ft. Make footing 4 ft wide.

DESIGN OF LINTELS 3rd Story.

$l = 19'$

Load from floor panel 16 x 4 @ 130#	=	9,800#
" " wall direct	=	4,700#
Weight of beam assuming 9" x 12"	=	2,100#
Total load	=	16,600#

M = 378,480" lbs Using 2% steel. C = 178.

$\therefore$ b = 9 in. d = 15"

3 - 1" corrugated bars. A_c = 3.08 sq. in.

Shear in these members is amply provided for in the actual section area.

Lintel : Bs'mt 1st and 2nd Stories:

These lintels will carry approximately the same loading as those of 3rd story therefore same dimensions will answer. However, the lintels over end window openings have only a 15' span. and may be reduced to 8"

x 14" having same reinforcement.

Design of Beam (B.) as per floor plan.

Span = 9' clear.

Uniform load 300 lbs. per sq. ft.

Load from floor direct	= 5,400#
" " Beam C.	= 8,000#
Weight of beam assuming 8 x 12	= 900#
Total	=14,300#

Assume $p = .02$ $c = 178.$

$M = cbd^2$ $d = 12"$

$$M = \frac{wl}{10} = \frac{14300 \times 9 \times 12}{10} = 154{,}400" \text{ lbs.}$$

$b = 8"$ $d = 13."$

A_s = 1o4 sq. in. A_c = 2.1 sq. in.

Use 3 - 7/8" Bars. and 3/8" stirrups spaced 8" c to c for 3 feet, then @ 12" spaces. Place 2-3/4" Bars over Beam (A) and also into lintel.

Beam A will carry the load from B of 7,150# without extra reinforcement as the amount is none above that deducted by the elevator shaft. Any excess will be easily carried by the factor of safety and again it is seldom that the total load will be thrown onto the floor at once. The shear in the above beam is amply provided for by the reinforcement.

DESIGN OF BEAM D around stair well.

Max. floor load = 300# . Per sq. ft. Clear span = 15'

Load from two beams. framing into D = 300 x 10 x 10.6 = 15,900 # Weight of beam @ 8" x 12" = 1400#

Total load = 17,300# Consider the load uniform

then $\frac{wl}{10} = M.$

M = 309600 " lbs. p = .03 b = 9"

C = 202 d = 14"

A_s=126 sq. in. A_c = 3.8 sq. in.

Use 4 - 1" bars.

Stirrups, use 3/8" twisted rods spaced 10" c to c to 4 feet from ends then @ 12" c to c. then place 2-3/4" Rods 6' long over supporting beams.

Design of Beam's (C). and (E).

Clear span = 10' p = .02 c = 178.

Load from floor @ 300# per sq. ft. = 14,300#
Assumed weight of beam = 1,000#
Total load 15,300#

M = 183600 " lbs.

Assume d = 13" then b = 8"

A_s = 104 sq. in. A_c = 2.1 sq. in.

Reinforce with 3- 7/8" bars.

Place 2 - 3/4" Bars over end, bending rods down over beam (B.).

Stirrups spaced same as in beam (B).

The shear is practically negligable any excess will be carried by the extra rods over beam.

Chapter 3.

General Specifications governing erection of proposed Building.

Proposition.

The Machine shop building to be constructed with reinforced concrete skeleton and faced with the best quality vitrified building brick. Sizes and demensions of members to be constructed as shown on the general Plans attached herewith.

Contract.

The Contractor will be required to sign the uniform Contract as adopted and recommended for general use by the American Institute of Architects and the National Association of Builders, the same is herewith attached. All of the articles of this contract will be considered as part and parcel of said contract.

The Engineers drawings and these specifications are intended to be supplementary to each other, and specifications given on the drawings shall have as full force as if given in the specifications, and in case of conflict between the two, specifications as given on the drawings shall be taken as superseding these specifications.

Excavation.

The contractor shall be required to make all necessary excavation, for walls, piers and footings to the proper depth as

shown on the plans. He shall remove all excess material excavated from the site of the structure, and shall be held liable for injuries to adjoining property during such excavation.

Materials.

The cement used throughout the construction shall be American Portland Cement of a well-known and tried brand. Such cement to be purchased by the Contractor, subject to the standard tests of the American Society for Testing Materials. Tests shall be made from each carload of cement and no cement shall be used on the job until such tests shall have been made.

Stone for massive concrete work, such as piers and foundation walls, shall preferably be crushed hard sand or other fire resisting stone in graded sizes of 1/4 to 2 1/2 inches.

Stone for reinforced concrete work shall be of similar material, but in graded sizes of from 1/4 to 1 inch. For very large masses of reinforced concrete such as the 1st and 2nd floor Girders, stone up to 1 1/2 inches may be used, and for very small work 3/4 inch shall be the maximum size.

Sand shall be clean, hard and free from earth or loam or other foreign matter, and shall be subject to the approval of the Architect. All stone shall be screened, unless otherwise distinctly arranged with the Architect or Engineer.

Steel for reinforced concrete shall be medium open hearth steel, manufactured in accordance with the Manufacturer's Standard and Specifications for this grade of material. The reinforcement in the floor slabs and footing shall be the regular

Ransome twisted rods as shown on the plans. The reinforcement in the beams, and girders shall be the regular Johnson corrugated bars. In all cases the steel shall have an ultimate strength of from 60,000 to 70,000 lbs. per sq. inch; and an elastic limit of not less than one-half the ultimate strength. Such steel shall not contain over .08 per cent of phosphorus.

Steel for reinforcing concrete will not be painted, but must be free from grease, dirt, or deep rust when placed in the work. In order to prevent rusting, steel bars must be protected from the weather whenever stored for over two weeks.

Forms.

All forms for moulded concrete work will be constructed of sufficient strength to obtain the necessary rigidity to prevent any motion of the forms while concrete is being placed and must be strong enough to carry any load which may come upon the concrete within 30 days from the date of placing the concrete.

All forms shall be so constructed as to be readily accessible for inspection at all places and at all times.

Forms shall in general be composed of tongue and grooved sheeting surfaced on two sides and supported by studding and braces of proper sizes. They will be placed in position by experienced and capable carpenters only and must be true to line and grade and of first-class workmanship throughout.

T and G sheeting will not be required for heavy work but carefully matched stuff may be used instead.

The forms for all exposed surfaces of concrete must be smooth and the boards must be carefully matched. The surface next the concrete will be oiled and forms used a second time must be carefully cleaned and oiled again.

The oil to be used in oiling forms shall be thick and heavy enough to act as a filler for the forms. The quality of oil known as "sludge" at the oil refineries or its equal is recommended.

Great care must be taken to clean all sawdust, dirt or debris from forms just before placing concrete, and whenever necessary forms must be cleaned out by steam jet or equally effective means.

All forms shall be so constructed as to be readily cleaned.

Forms shall not be removed from the under side of floor slabs in less than 10 and preferably less than 14 days; shores from beneath girders and beams in less than 14 days and the forms shall always be removed from the columns and inspection made of their condition before removing the shores from beams and girders. The above limits apply to work done in warm weather from April 1st to December 1st. For work done in winter from December 1st to April 1st forms shall not be removed in less than 1 1/2 the time as specified above.

But no forms whatever will be removed at any time without first notifying the Architect or the Engineer in charge. But such notification shall not be considered to relieve the Contractor of responsibility for the construction and for the removal of such forms.

All forms must be designed so that they may be removed with as little damage as possible to the concrete or to the forms.

All projecting wires, bolts, or other devices used for securing forms and that pass through the concrete must be cut off at least one inch beneath the finished surface and the ends covered with cement mortar. The purpose of this is to prevent discoloration of the concrete by corrosion of the steel.

Forms for columns shall be constructed of sheet steel not less than 1/4 inch in thickness, and so manufactured as to be water tight and with not more than two joints along the column.

Anchorages etc.

The contractor for concrete work must place all anchor bolts for columns, beams etc., and must place all anchors and ties for all attachments to the concrete work, such as trimmings and brick facings.

All such bolts and anchors will be furnished by the contractor.

The contractor shall use every precaution to place the reinforcing rods or bars in the position shown on the plans, and during the placing of concrete, he shall be expected to see that such rods or bars are not disturbed or moved from their proper position.

Surface Finish.

There will be no mortar surface used in this work, but great care must be taken to insure the flushing of the cement against the forms. This shall be done by careful spading and puddling of the plastic concrete while being placed in the forms.

Concrete Floors and Wearing Surfaces.

The basement floor which will be laid on the ground will be laid on a bed of cinders or gravel or sand carefully compacted and at least six (6) inches in thickness. On top of the foundation layer shall be laid a binder course of concrete not less than 3 1/2 inches in thickness. Such layer of concrete to be

composed of one part cement to three parts sand and six parts stone. On top of the binder course and before the same has set, will be placed 1/2 inch of cement mortar, composed of one part Portland cement to two parts of hard selected crushed granite. This finish will be screeded with a straight edge to grade, smoothed with a wooden float, finished with a steel trowel and marked off into blocks of suitable size.

Reinforced concrete floors, will be provided with a wearing surface proportioned and placed in the same manner as the finish coat for the basement floor, but in this case the floors will usually be finished after the forms for the concrete of the upper floors have been removed.

If an efficient bond can be obtained between the old concrete and the finish coat, by use of an adhesive preparation, the finish coat may be only 3/4 inch thick; but the Contractor must in such case, give satisfactory gaurantee of the efficiency of the bond.

Otherwise, the finish coat must be at least 1 1/2 inches in thickness, and the bond between the old concrete and the new must be made by carefully cleaning and wetting the surface of the former and sprinkling with wet cement just before applying the finish coat.

If the surface can be placed before the floor has set, only 1/2 inch of finish mortar will be required.

Freezing Weather:-

No concrete will be laid in freezing weather except by spec-

ial arrangement with and under the supervision of the Engineer in charge. In case it becomes necessary to lay concrete in freezing weather, special arrangements must be made for heating all materials and maintaining a temperature which will not allow the concrete to freeze until the same has set.

Proportioning, Mixing and Placing.

All concrete proportions specified herein will be based upon the assumption that one barrel of Portland cement is equivalent to 3.8 cubic feet, and all proportioning must be done by means of a carefully gauged wheelbarrow or other apparatus of capacity which will be determined by the Architect or Engineer.

The concrete used throughout the structure with the exception of piers and foundation walls, shall be composed of one part Portland cement to two parts sand to four parts of stone, such materials to answer the requirements as herein specified.

All mixing shall be by machinery except such hand mixing as may be allowed by special arrangement with the Architect or Engineer.

If concrete is mixed by hand the sand and cement shall be spread upon the mixing board in thin layers and turned with spades until the mixture is of uniform color. Stone and water shall then be added, and the mass shall then be turned at least three times, not counting the shoveling off the board.

Preference will be given to revolving batch machines which automatically measure the ingredients of the mix. Mixing must be very carefully and thoroughly done. Enough water must be

used to make the mass plastic enough to run freely, in other words, a wet mix will be required for all reinforced concrete work. For massive concrete work, only enough water will be used to make the concrete plastic; not so wet but that it may be churned with a light tamper sufficient to quake the mass.

The concrete shall be placed in position immediately after mixing and before the initial set shall have taken place.

No retempering of concrete which has been allowed to stand until the initial set has taken place, will be allowed unless by special permission of the Architect or Engineer.

The mixing and placing of concrete will be as far as practicable a continuous operation, and when it is necessary to make a joint, it shall be made in the middle of a panel at right angles to the beams and by means of a stop board placed in a vertical position and containing a key on the side next to the block first placed. When these stop boards are removed, the exposed surfaces of concrete shall be wet and carefully dusted with neat cement or painted with mortar before continuing the next block.

Concrete must be kept wet for one week after depositing, and in dry hot weather must in addition be kept from exposure to the sun during this time, or arrangements be made for constant sprinkling.

Waterproofing.

The roof surface shall be waterproofed in the following manner.

The surface to be coated shall be painted with asphalt reduced with naptha, then a thin layer of hot asphalt shall be applied and thoroughly mopped over, then a coat of asphalt and clean dry sand in the proportion of one part asphalt to three parts sand by volume; this coat to be thoroughly mixed in the kettle and spread on with warm smoothing irons; the finishing coat to consist of pure hot asphalt spread thinly and evenly over the surface and then sprinkled with gravel. The finished coat shall be about 3/4 inch thick.

All asphalt shall be of the best grade, free from coal tar or coal tar products.

It must not volatilize more than one-half of one per cent under a temperature of 300° Fahr. for 10 hours. It must not be affected by 20 per cent solution of ammonia; 35 per cent solution of hydrochloric acid; 25 per cent solution of sulphuric acid nor by a saturated solution of sodium chloride. It must not flow under 185° Fahr. nor become brittle above 0° Fahr.

General Conditions and Requirements.

The Contractor shall furnish a bond in the sum of one-fourth of the amount of his contract with a satisfactory surety company as surety that the contract will be fulfilled.

Should their be any discrepancy between the drawings and specifications, between scale or full size drawing and between scale of the drawings and figures on the same, the matter shall be referred to the Architect or Engineer, and his decision shall be followed.

The Architect and Engineer will give all datum lines and levels necessary for the prosecution of the work and be responsible for the accuracy thereof.

The Contractor shall give to the proper authorities all requisite notices relating to the work in his charge and shall conform to all laws and ordinances affecting same, shall obtain all official permits and licenses required and pay all proper fees for the same, and he shall pay for any other legal charges from city, county, state or federal officers.

The work shall be suitably protected by red cautionary lights at night and by watchmen if necessary.

The execution of the work will be entirely at the Contractor's risk and he will be liable for its safety. The Contractor will be liable for all cases of personal injury which may occur during the progress of the work.

All royalties for patents or claims for the infringements thereof that may be involved in the construction of this work shall be included in the contract amount and the contractor shall satisfy all demands that may be made at any time for such royalties and be liable for any damages or claims for infringements of patents.

At periods not less than 30 days apart on the request of the Contractor, the Architect or Engineer will make estimates of the value of material furnished and labor performed, and the amount of such estimates less 10 per cent will be paid by the contractor by the owner withing 30 days after such estimate has beenpre-

sented for payment. The final payment will be made in 60 days after the completion of the work and the acceptance thereof by the Architect or engineer on behalf of the owner.

On completion of the work the Contractor shall remove from the site all debris and rubbish at such times as may be directed by the Architect or Engineer.

The Architect or Engineer will have an Inspector on this work, whose duties will be to see that these specifications, the plans and contract are faithfully fulfilled, and that all work is of a strictly first-class character.

The Inspector will have no jurisdiction over the workmen, but he will report all violations of the specifications, plans, or contract, or any departure from strictly first-class work, to the Contractor or his highest representative, verbally, and to the Architect or Engineer in writing.

If such defective work be not made good immediately after such verbal notification on the part of the Inspector, then the Architect or Engineer will give written instructions to the Contractor to make same good, and any violation or disregard of such written instructions on the part of the Contracotr will be considered to be a violation of the contract, (Subject to the Arbitration Clause of the Uniform Contract.)

The inspector will be paid by the Architect, Engineer or Owner and will receive no compensation from the Contractor.

Shear Diagrams for 1st & 2nd Fl. Beams and Girders.

FORM 19642-S.

THE

UNIFORM CONTRACT.

FORM OF CONTRACT

ADOPTED AND RECOMMENDED FOR GENERAL USE

BY THE

AMERICAN INSTITUTE OF ARCHITECTS

AND THE

NATIONAL ASSOCIATION OF BUILDERS.

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REVISED [illegible]

This Agreement, made the ________ day of ________ in the year one thousand nine hundred and ________ by and between

____

________ party of the first part (hereinafter designated the Contractor), and

____

________ party of the second part (hereinafter designated the Owner),

Witnesseth that the Contractor , in consideration of the agreements herein made by the Owner , agree with the said Owner as follows:

ARTICLE I. The Contractor shall and will provide all the materials and perform all the work for the

____

________ as shown on the drawings and described in the specifications prepared by

________ Architect, which drawings and specifications are identified by the signatures of the parties hereto, and become hereby a part of this contract.

ART. II. It is understood and agreed by and between the parties hereto that the work included in this contract is to be done under the direction of the said Architect, and that his decision as to the true construction and meaning of the drawings and specifications shall be final. It is also understood and agreed by and between the parties hereto that such additional drawings and explanations as may be necessary to detail and illustrate the work to be done are to be furnished by said Architect, and they agree to conform to and abide by the same so far as they may be consistent with the purpose and intent of the original drawings and specifications referred to in Art. I.

It is further understood and agreed by the parties hereto that any and all drawings and specifications prepared for the purposes of this contract by the said Architect are and remain his property, and that all charges for the use of the same, and for the services of said Architect, are to be paid by the said Owner .

ART. III. No alterations shall be made in the work except upon written order of the Architect; the amount to be paid by the Owner or allowed by the Contractor by virtue of such alterations to be stated in said order. Should the Owner and Contractor not agree as to amount to be paid or allowed, the work shall go on under the order required above, and in case of failure to agree, the determination of said amount shall be referred to arbitration, as provided for in Art. XII of this contract.

ART. IV. The Contractor shall provide sufficient, safe and proper facilities at all times for the inspection of the work by the Architect or his authorized representatives; shall, within twenty-four hours after receiving written notice from the Architect to that effect, proceed to remove from the grounds or buildings all materials condemned by him, whether worked or unworked, and to take down all portions of the work which the Architect shall by like written notice condemn as unsound or improper, or as in any way failing to conform to the drawings and specifications, and shall make good all work damaged or destroyed thereby.

ART. V. Should the Contractor at any time refuse or neglect to supply a sufficiency of properly skilled workmen, or of materials of the proper quality, or fail in any respect to prosecute the work with promptness and diligence, or fail in the performance of any of the agreements herein contained, such refusal, neglect or failure being certified by the Architect, the Owner shall be at liberty, after three days written notice to the Contractor , to provide any such labor or materials, and to deduct the cost thereof from any money then due or thereafter to become due to the Contractor under this contract; and if the Architect shall certify that such refusal, neglect or failure is sufficient ground for such action, the Owner shall also be at liberty to terminate the employment of the Contractor for the said work and to enter upon the premises and take possession, for the purpose of completing the work included under this contract, of all materials, tools and appliances thereon, and to employ any other person or persons to finish the work, and to provide the materials therefor; and in case of such discontinuance of the employment of the Contractor ...... shall not be entitled to receive any further payment under this contract until the said work shall be wholly finished, at which time, if the unpaid balance of the amount to be paid under this contract shall exceed the expense incurred by the Owner in finishing the work, such excess shall be paid by the Owner to the Contractor ; but if such expense shall exceed such unpaid balance, the Contractor shall pay the difference to the Owner . The expense incurred by the Owner as herein provided, either for furnishing materials or for finishing the work, and any damage incurred through such default, shall be audited and certified by the Architect, whose certificate thereof shall be conclusive upon the parties.

ART. VI. The Contractor shall complete the several portions, and the whole of the work comprehended in this Agreement by and at the time or times hereinafter stated, to wit:

ART. VII. Should the Contractor be delayed in the prosecution or completion of the work by the act, neglect or default of the Owner , of the Architect , or of any other contractor employed by the Owner upon the work, or by any damage caused by fire or other casualty for which the Contractor ....not responsible, or by combined action of workmen in no wise caused by or resulting from default or collusion on the part of the Contractor , then the time herein fixed for the completion of the work shall be extended for a period equivalent to the time lost by reason of any or all the causes aforesaid, which extended period shall be determined and fixed by the Architect ; but no such allowance shall be made unless a claim therefor is presented in writing to the Architect within forty-eight hours of the occurrence of such delay.

ART. VIII. The Owner agree to provide all labor and materials essential to the conduct of this work not included in this contract in such manner as not to delay its progress, and in the event of failure so to do, thereby causing loss to the Contractor , agree that .... will reimburse the Contractor for such loss; and the Contractor agree that if ....shall delay the progress of the work so as to cause loss for which the Owner shall become liable, then............shall reimburse the Owner for such loss. Should the Owner and Contractor fail to agree as to the amount of loss comprehended in this Article, the determination of the amount shall be referred to arbitration as provided in Art. XII of this contract.

ART. IX. It is hereby mutually agreed between the parties hereto that the sum to be paid by the Owner to the Contractor for said work and materials shall be ....

subject to additions and deductions as hereinbefore provided, and that such sum shall be paid by the Owner to the Contractor , in current funds, and only upon certificates of the Architect, as follows:

(Form 19642-S.)

The final payment shall be made within .... days after the completion of the work included in this contract, and all payments shall be due when certificates for the same are issued.

If at any time there shall be evidence of any lien or claim for which, if established, the Owner of the said premises might become liable, and which is chargeable to the Contractor , the Owner shall have the right to retain out of any payment then due or thereafter to become due an amount sufficient to completely indemnify.................against such lien or claim. Should there prove to be any such claim after all payments are made, the Contractor shall refund to the Owner all moneys that the latter may be compelled to pay in discharging any lien on said premises made obligatory in consequence of the Contractor default.

Art. X. It is further mutually agreed between the parties hereto that no certificate given or payment made under this contract, except the final certificate or final payment, shall be conclusive evidence of the performance of this contract, either wholly or in part, and that no payment shall be construed to be an acceptance of defective work or improper materials.

Art. XI. The Owner shall during the progress of the work maintain insurance on the same against loss or damage by fire,.. the policies to cover all work incorporated in the building, and all materials for the same in or about the premises, and to be made payable to the parties hereto, as their interest may appear.

Art. XII. In case the Owner and Contractor fail to agree in relation to matters of payment, allowance or loss referred to in Arts. III or VIII of this contract, or should either of them dissent from the decision of the Architect referred to in Art. VII of this contract, which dissent shall have been filed in writing with the Architect within ten days of the announcement of such decision, then the matter shall be referred to a Board of Arbitration to consist of one person selected by the Owner , and one person selected by the Contractor , these two to select a third. The decision of any two of this Board shall be final and binding on both parties hereto. Each party hereto shall pay one-half of the expense of such reference.

The said parties for themselves, their heirs, successors, executors, administrators and assigns, do hereby agree to the full performance of the covenants herein contained.

In Witness Whereof, the parties to these presents have hereunto set their hands and seals, the day and year first above written.

In Presence of

THE

UNIFORM CONTRACT

FORM OF CONTRACT
ADOPTED AND RECOMMENDED FOR GENERAL USE BY THE
AMERICAN INSTITUTE OF ARCHITECTS
AND THE
NATIONAL ASSOCIATION OF BUILDERS.
REVISED 1906 AND 1907.

AGREEMENT

BETWEEN

Contractor ,

AND

Owner ,

FOR

190

ARCHITECT

AMOUNT OF CONTRACT

$

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125 EAST 42D STREET, NEW YORK, N. Y.
(Form 19642-S.)

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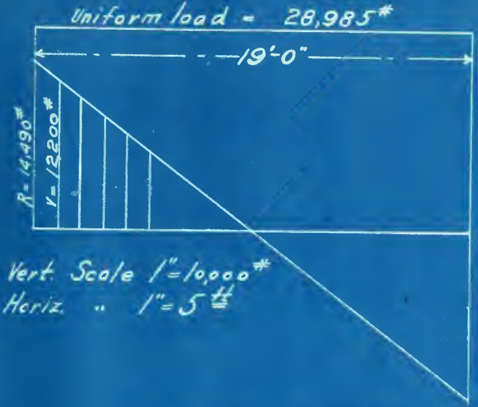
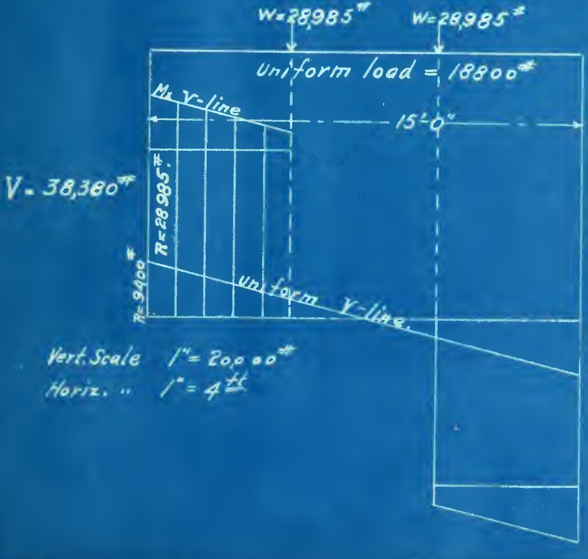
(Form 19642-S.)

Shear Diagrams for 1st + 2nd Fl. Beams and Girders.



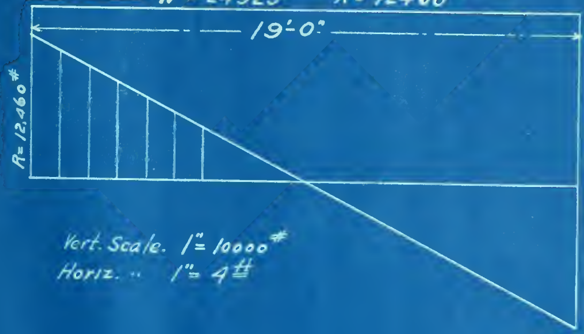
Floor beams 1-2-A



Floor Girders 1-2-B

Shear Diagrams for 3rd Fl. beams & Girders.

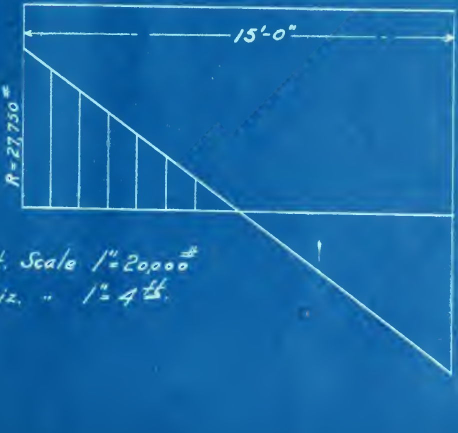
W = 24925# R = 12460#



Vert. Scale. 1" = 10000#
Horiz. " 1" = 4 ft

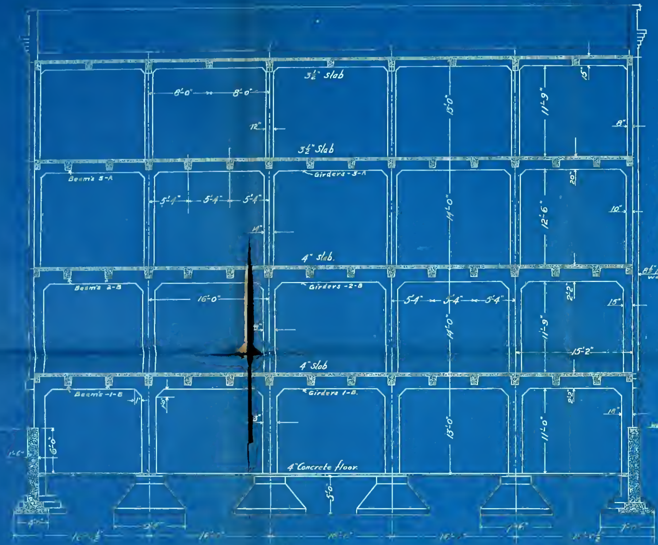
Floor beams
3-A

W = 55,510#



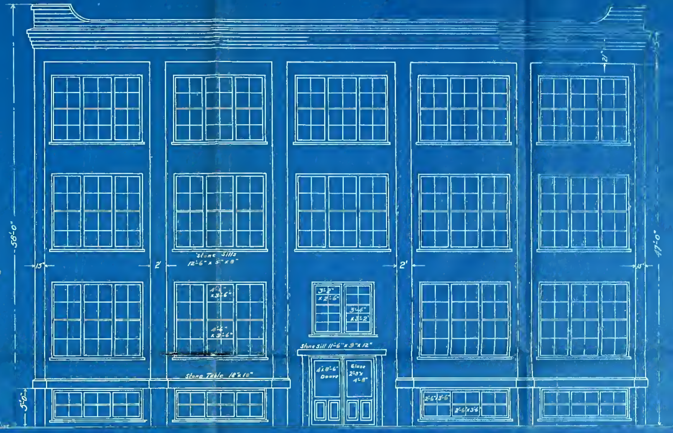
Vert. Scale 1" = 20,000#
Horiz. " 1" = 4 ft.

Floor Girders
3-A



SECTION "AB" SHOWING ARRANGEMENT OF BEAMS, GIRDERS & COLUMNS

Scale ¼"=1ft.



END ELEVATION

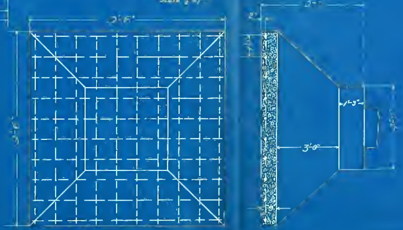
Scale ⅛"=1ft.

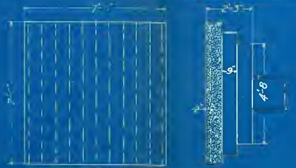
DETAIL OF COLUMN FOOTINGS

⅝" Twisted rods 12" c to c.

1-3-6 Concrete

Scale ½"=1ft.





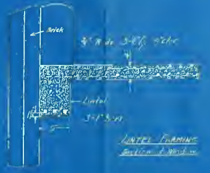
DETAIL OF WALL COLUMN FOOTINGS

⅝" Twisted bars transverse

to wall, bars 8" c to c

1-3-6 Concrete

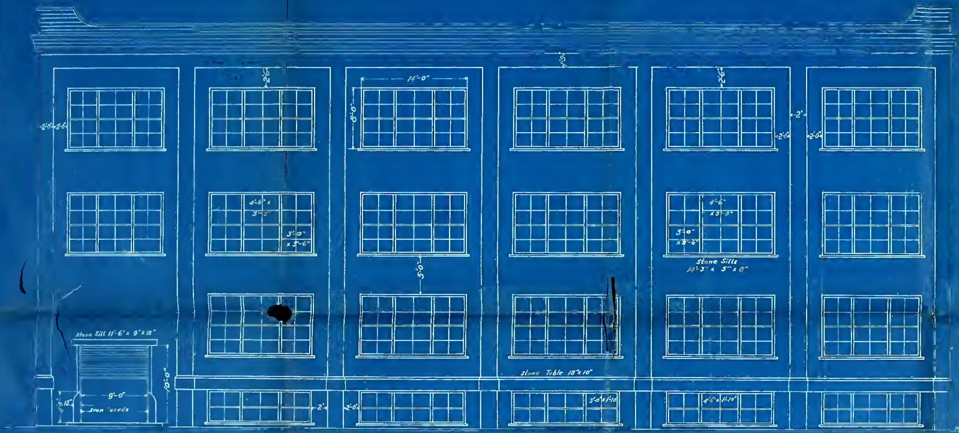
Scale ½"=1ft.



LINTEL FRAMING

Section of Window

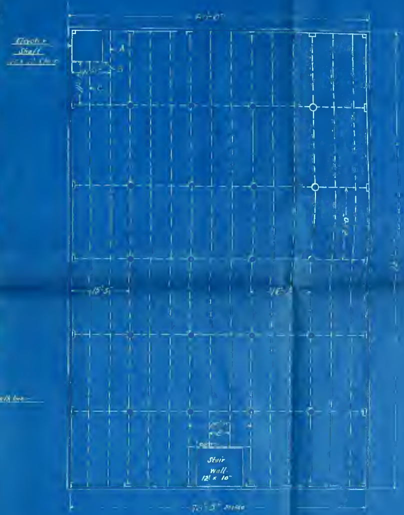
ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
PLANS FOR A 3-STORY
MACHINE SHOP BUILDING
— THESIS OF —
Harold S. Ellington May 25-1912
Sheet No. 1.



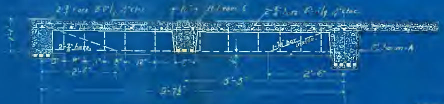
Entrance to Elevator
Rolling Sheet Steel Door

Same door on other side
plus window frame similar
window each on far side

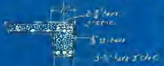
SIDE ELEVATION
Wall facing of Vitrified brick
Scale 1/8"=1'



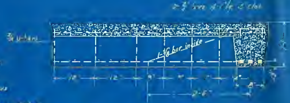
GENERAL FLOOR PLAN



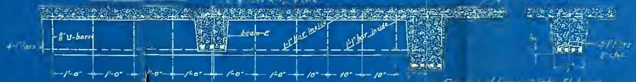
Details of Beams-B Showing framing of Lintels and Fl-beam-A
Same for All Floors.



Section of beam-D



Framing of Beam-C into Beam-B
Beam E Same details



Framing of Beam-D into Fl. beams

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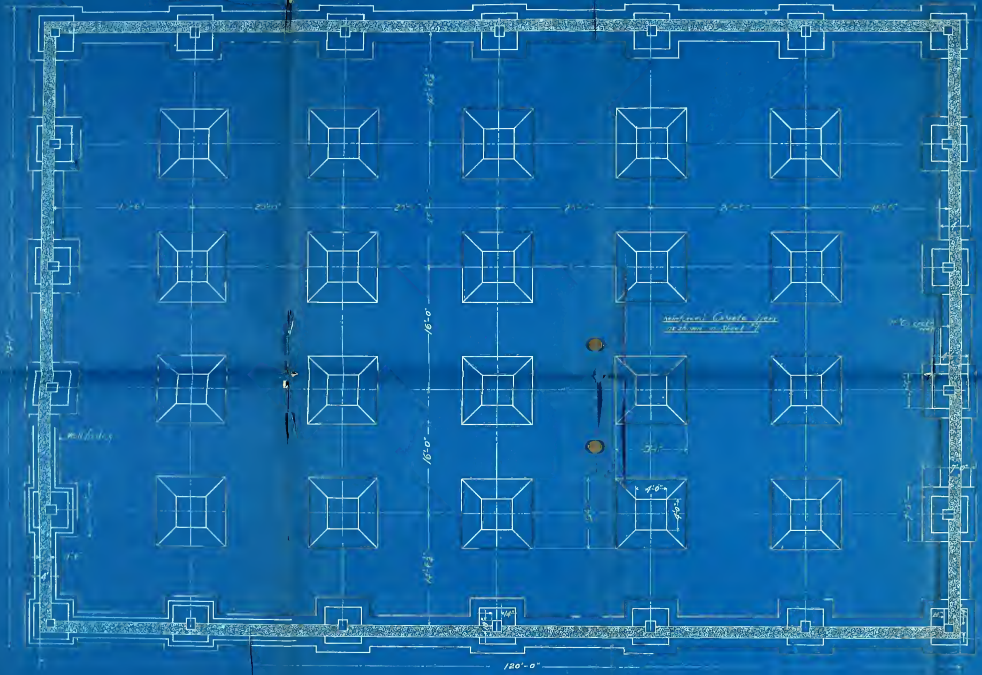
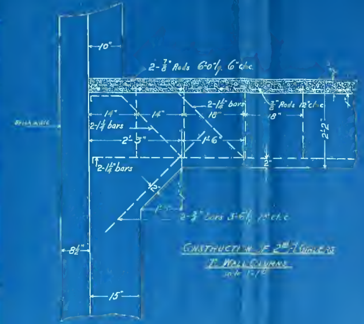
CIVIL ENGINEERING DEPARTMENT
PLANS FOR A 3-STORY
MACHINE SHOP BUILDING
— THESIS OF —
Harold S. Ellington May, 25-1908
Sheet No. 2.

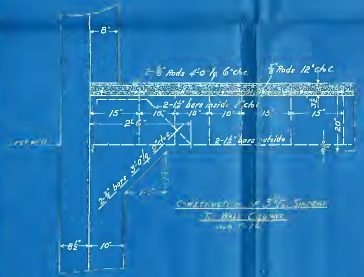


GENERAL PLAN OF FOUNDATIONS.

All footings and walls 1-3-6 Concrete
Scale 1/8" = 1 ft.

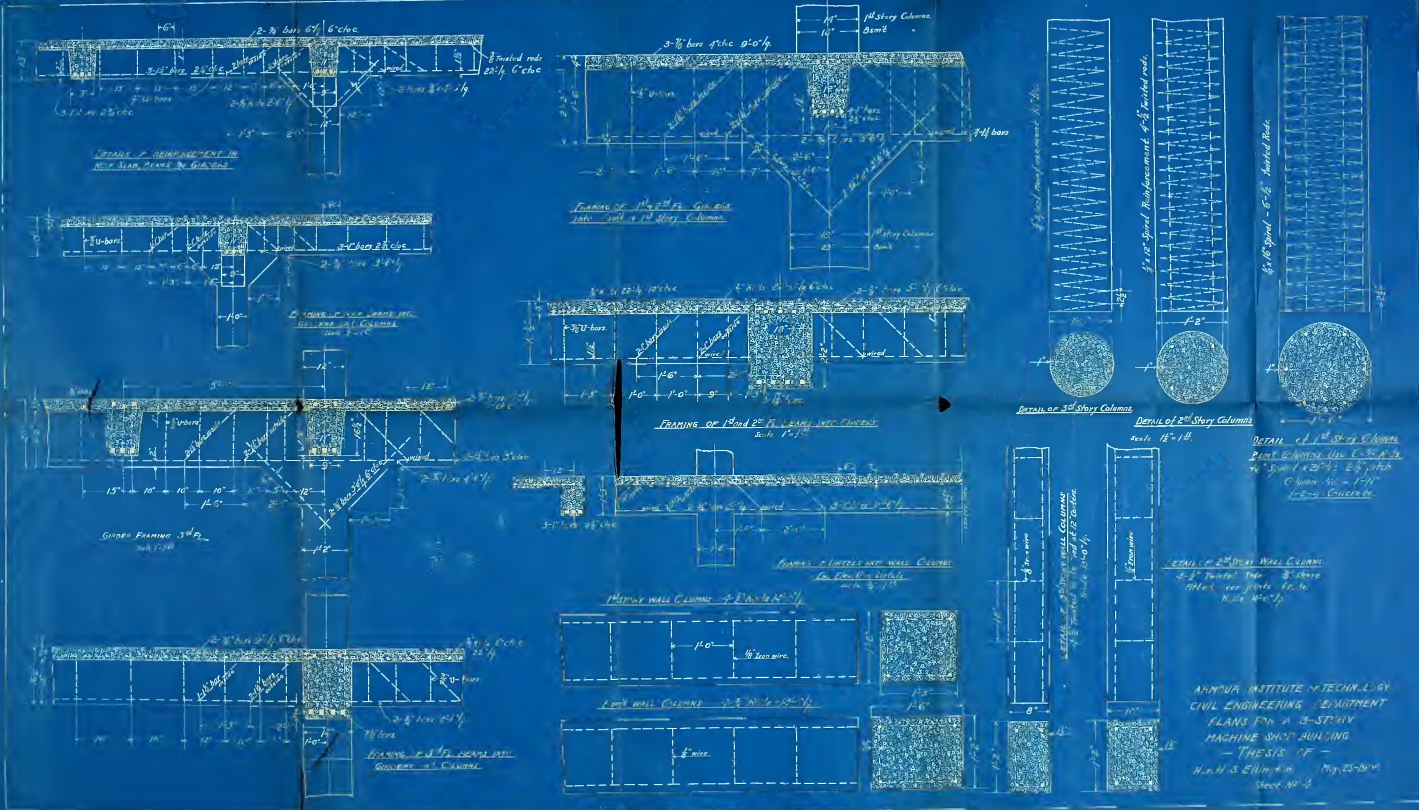


CONSTRUCTION OF 2ND FL. GIRDERS TO WALL COLUMNS
Scale 1" = 1 ft.


8"
2-7/8" Rods 4'-0" lg. 6" c. to c.
5/8" Rods 12" c. to c.
2-1½" bars inside 3" c. to c.
15"
10"
10"
10"
15"
15"
3½"
2'-6"
20"
2-1½" bars outside
2"
2-3/4" bars 5'-0" lg. 9" c. to c.
Brick wall
8½"
10"

CONSTRUCTION OF 3RD FL. GIRDERS TO WALL COLUMNS
Scale 1" = 1 ft.

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
PLANS FOR A 3-STORY
MACHINE SHOP BUILDING.
— THESIS OF —
Harold S. Ellington   May, 25-1908.
Sheet No. 8.


Details of Reinforcement in Roof Slab, Beams & Girders
Framing of Roof Beams into Girders and Columns
Scale 1"=1'
Girder Framing 3rd Fl.
Scale 1"=1'
Framing of 3rd Fl. Beams into Girders and Columns
Framing of 1st & 2nd Fl. Girders into Bsmt & 1st Story Columns
Framing of 1st and 2nd Fl. Beams into Girders
Scale 1"=1'
Framing of Lintels into Wall Columns
End Elevation Lintels
1st Story Wall Columns
Bsmt Wall Columns
Detail of 3rd Story Columns
Detail of 2nd Story Columns
Scale 1½"=1 ft.
Detail of 1st Story Columns
Bsmt Columns Use 6-3⁄4" Rods
Detail of 2nd Story Wall Columns
1st Story Columns
Bsmt
3-7/8" bars 4" c to c 9'-0" l/g.
2-3/4 bars 6' l/g 6" c to c
3-1/2" bars 2½" c to c
3/8" Twisted rods 22' l/g 6" c to c
3/8" U-bars
3-1" bars 2½" c to c
2-1" bars inside
2-1" bars outside
wired
4-1½ bars
1/8" Iron wire.
1/8" wire
¼" x 12" Spiral Reinforcement 4-1½" Twisted rods.
¼" x 16" Spiral - 6-1½" Twisted Rods.
1'-0"
1'-6"
1'-2"
8"
10"
12"
18"
2½"

ARMOUR INSTITUTE OF TECHNOLOGY
CIVIL ENGINEERING DEPARTMENT
PLANS FOR A 3-STORY
MACHINE SHOP BUILDING
— THESIS OF —
Harold S. Ellington May 25-1908
Sheet No 2